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Indian Standard

METHODS FOR
FIXING THE CAPACITIES OF RESERVOIRS

PART IV FLOOD STORAGE

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METHODS FOR
FIXING THE CAPACITIES OF RESERVOIRS

PART IV   FLOOD STORAGE

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Indian Standard

METHODS FOR
FIXING THE CAPACITIES OF RESERVOIRS
PART IV FLOOD STORAGE

0. FOREWORD

0.1 This Indian Standard (Part IV) was adopted by the Indian Standards Institution on 19 February 1971, after the draft finalized by the Catchment Area and Reservoirs Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Flood storage depends on the height at which the maximum water level (MWL) is fixed above the normal conservation level (NCL). The determination of the MWL involves the routing of the design flood through the reservoir and spillway. When the spillway capacity provided is low, the flood storage required for moderating a particular flood will be large and *vice versa*. A higher MWI involves larger submergence and hence this aspect has also to be kept in view while fixing the MWL and the flood storage capacity of the reservoir.

0.3 This standard consists of four parts and the other parts are as follows:

IS:5477(Part I)-1969 Methods for fixing the capacities of reservoirs: Part I General requirements

IS:5477(Part II)-1969 Methods for fixing the capacities of reservoirs: Part II Dead storage

IS:5477 (Part III)-1969 Methods for fixing the capacities of reservoirs: Part III Live storage

0.4 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analy·   shall be rounded off in accordance with IS:2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard (Part IV) covers the criteria and procedure to be followed in fixing the flood storage capacity of a reservoir consistent with the

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*Rules for rounding off numerical values (*revised*).

safety of the structure itself and the life and properties downstream of the reservoir.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

2.1 Normal Conservation Level (NCL) — The normal conservation level is the highest level of the reservoir at which water is intended to be held for various uses other than flood control.

2.2 Full Reservoir Level (FRL) — Is the highest level of the reservoir at which water is intended to be held for various uses including part or total of the flood storage without allowing any passage of water through the spillway.

2.3 Maximum Water Level (MWL) — Is the highest level to which the reservoir waters will rise while passing the design flood with the spillway facilities in full operation.

2.4 Surcharge Storage — The storage between the crest of an uncontrolled spillway or the top of the crest gates in normal closed position and the MWL is termed as the surcharge storage.

2.5 Maximum Probable Flood — Is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region, and is computed by using the maximum probable storm which is an estimate of the physical upper limit to storm rainfall over the basin. This is obtained from storm studies of all the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions (*see* **A-2.1.1** of Appendix A also).

2.6 Standard Project Flood — Is the flood that may be expected from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the region and is computed from the standard project storm rainfall reasonably capable of occurrence over the basin in question and may be taken as the largest storm which has occurred in the region of the basin during the period of weather record. It is not maximized for most critical atmospheric conditions but it may be transposed from an adjacent region to the water-shed under consideration.

2.7 Design Flood — Is the flood adopted for design purposes. It may be the maximum probable flood or the standard project flood or a flood corresponding to some desired frequency of occurrence depending upon the standard of security that should be provided against possible failure of the structure.

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3. GENERAL

3.1 A prerequisite essential to the safety of the dam is a decision in regard to the standard of security that should be provided against possible failure of the concerned structure during extraordinary floods. In case, failure of the structure is likely to result in loss of life or widespread property damage in addition to the damage of the structure itself as in the case of spillways of large reservoirs, a very high degree of security shall be provided against failure under the most severe flood conditions considered reasonably possible. But when loss of life is not involved, economic considerations would prevail to govern the selection of the design flood.

3.2 It should be assumed that the reservoir would be filled to the full reservoir level at the beginning of the spillway design flood. Even though the contemplated plan of reservoir operation indicates that a portion of the storage capacity below the FRL probably would be available at the beginning of the spillway design flood, the possibility of improper operation of regulating outlets as the result of incorrect flood predictions, mechanical difficulties, plugging of conduits by debris, or negligent attendance may justify the assumption of a full reservoir at the beginning of the design flood. Moreover, future developments may require revisions in the original plan of reservoir operation or changes in the use of the reservoir that would increase the probability of a full reservoir at the beginning of the spillway design flood.

4. ESTIMATION OF DESIGN FLOOD

4.1 The methods in vogue for estimation of design flood are broadly classified as under:

    a) Application of a suitable factor of safety to maximum observed flood or maximum historical flood,

    b) Empirical flood formulae,

    c) Envelope curves,

    d) Frequency analysis, and

    e) Rational method of derivation of design flood from storm studies and application of unit hydrograph principle.

4.2 Application of a Suitable Factor of Safety to Maximum Observed Flood or Maximum Historical Flood — The design flood is obtained by applying a safety factor which depends upon the judgement of the designer to the observed or estimated maximum historical flood at the project site or nearby site on the same stream. This method is limited by the highly subjective selection of a safety factor and the length of available stream flow record which may give a quite inadequate sample of flood magnitudes likely to occur over a long period of time.

4.3 Empirical Flood Formulae — The empirical formulae commonly used in the country are the Dicken's formula, Ryve's formula and Inglis' formula in which the peak flow is given as a function of the catchment area and a coefficient. The values of the coefficient vary within rather wide limits and have to be selected on the basis of judgement. They have limited regional application, should be used with caution and only, when a more accurate method cannot be applied for lack of data.

4.4 Envelope Curves — In the envelope curve method maximum flood is obtained from the envelope curve of all the observed maximum floods for a number of catchments in a homogeneous meteorological region plotted against drainage area. This method, although useful for generalizing the limits of floods actually experienced in the region under consideration, cannot be relied upon for estimating maximum probable floods for the determination of spillway capacity except as an aid to judgement.

4.5 Frequency Analysis — The frequency method involves the statistical analysis of observed data of a fairly long (at least 25 years) period.

4.5.1 A purely statistical approach when applied to derive design floods for long recurrence intervals several times larger than the data has many limitations and hence this method has to be used with caution.

4.6 Rational Method of Derivation of Design Flood from Storm Studies and Application of Unit Hydrograph Principle

4.6.1 The steps involved, in brief, are:

a) analysis of rainfall *Vs* run-off data for derivation of loss rates under critical conditions;

b) derivation of unit hydrograph by analysis (or by synthesis, in cases where data are not available);

c) derivation of the design storm; and

d) derivation of design flood from the design storm by the application of the rainfall excess increments to the unit hydrograph.

Appendix A provides the criteria for estimation of design flood.

4.6.2 *Unit Hydrograph — Limitations*

a) The unit hydrograph principle is not applicable for drainage basins having an area of more than 5 000 km² where valley storage effects are not reflected and where variation of rainfall in space and time shows a tendency to become too great to be reflected in the unit hydrograph.

b) Application of the unit hydrograph principle is also not recommended for catchments having an area less than about 25 km².

c) Large number of raingauges suitably located should be available in. the entire catchment to reflect the true weighted rainfall of the catchment.

d) Unit hydrograph principle is not applicable when appreciable proportions of the precipitation occurs in the form of snow or when snow covers a significant part of the catchment.

5. DETERMINATION OF MAXIMUM WATER LEVEL

5.1 The maximum water level of a reservoir is obtained by routing the design flood through the reservoir and spillway. This process of computing the reservoir stages, storage volumes and outflow rates corresponding to a particular hydrograph of inflow is commonly referred to as flood routing. The routing is determined by the following :

a) Initial reservoir stage ;
b) The design-flood hydrograph ;
c) Rate of outflow including the flow over the crest, through sluices or outlets and through power units ; and
d) Incremental storage capacity (each stage).

5.1.1 Although there are definite relations between reservoir inflow, storage, and outflow these relations are usually difficult to express algebraically. Therefore, a step-by-step computation procedure is followed whereby the increase in storage and rate of outflow resulting from the volumes of inflow during successive short increments of time are computed. Increments of inflow are computed for periods of time sufficiently short to warrant the assumption that the mean of the inflow rates and the mean of the outflow rates at the beginning and end of the intervals would closely approximate the average rates for the respective periods.

5.2 Appendix B provides an illustration with the details of routing the inflow hydrograph through a storage reservoir and fixing the maximum water level. Appendix C gives an example based on graphical (Sorensen) method of routing the inflow hydrograph through a reservoir.

APPENDIX A
(Clauses 2.5 and 4.6.1)

CRITERIA FOR ESTIMATION OF DESIGN FLOODS FOR DAMS AND OTHER HYDRAULIC STRUCTURES

A-1. GENERAL

A-1.1 Each site is individual in its local conditions, and evaluation of causes and effects. Therefore, only general guidelines are provided and

the hydrologists and the designer would have the discretion to vary the criteria in special cases, where the same are justifiable on account of assessable and acceptable local conditions, which should be recorded and have the acceptance of the competent authority.

A-2. DETERMINATION OF DESIGN FLOOD

A-2.1 Major and Medium Reservoirs

A-2.1.1 *Maximum Probable Flood* — In the design of spillways for major and medium reservoirs (with storages more than 6000 hectare metres) the maximum probable flood should be used. The maximum probable flood is estimated from the maximum probable storm applying the unit hydrograph principle. The maximum probable storm is an estimate of the physical upper limit to storm rainfall over a basin. It is obtained from the studies of all the storms that have occurred in the region and maximizing them for possible moisture charge and for storm efficiency.

The method of moisture adjustment commonly used involves the estimation of air moisture content from surface level dew point observations. Maximizing storm efficiency is achieved by storm transposition which assumes that the most effective combination of storm efficiency and inflow wind has either occurred or has been closely approached in the outstanding storm on record. Further adjustment of storm precipitation data to the estimated maximum sustained wind to carry this maximum moisture supply into the project basin is not attempted unless very high design safety factors are desired or only very limited storm rainfall data are available.

A-2.1.2 *Distribution of Storm Rainfall* — The distribution of storm intensities for small durations is obtained on the basis of recorded data of self-recording raingauge stations in the catchment or region.

A-2.1.3 *Maximization for Unit Hydrograph Peak* — It is observed in practice that the unit hydrograph peak obtained from heavier rainfalls is about 25 to 50 percent higher than those obtained from the smaller rainfalls. Therefore, the unit hydrograph from the observed floods may have to be suitably maximized up to a limit of 50 percent depending upon the judgement of the hydrologist. In case the unit hydrograph is derived from very large floods, then the increase may be of a very small order; if it is derived from low floods the increase may have to be substantial.

A-2.1.4 *Loss Rate* — The loss rate should be estimated from the volume of observed flood run-off and the corresponding storm that caused the flood. A minimum loss rate should be worked out and adopted for computing the design flood.

A-2.2 The probability method, when applied to derive design floods for long recurrence intervals several times larger than the length of data, has

many limitations. In certain cases, however, like that of very large catchments where unit hydrograph method is not applicable and where sufficient long term discharge data is available, the frequency method may be the only course possible. In such cases the design flood to be adopted for major structures should have a frequency of not less than once in 1 000 years. Where annual flood values of adequate length are available, they are to be analyzed by Gumbel's method (*see* Appendix D), and where the data is short, either partial duration method or regional frequency technique is to be adopted as a tentative approach and the results verified and checked by hydrological approaches.

Sometimes when the flood data is inadequate, frequency analysis of recorded storms is made and the storm of a particular frequency applied to the unit hydrograph to derive the flood; this flood usually has a return period greater than that of the storm.

A-2.3 Barrages and Minor Dams — In the case of parmanent barrages and minor dams with less than 6 000 hectare metres storage, the standard project flood or a 100 year flood, whichever is higher, is to be adopted.

A-2.3.1 For pick-up weirs a flood 50-100 years frequency should be adopted according to its importance, and level conditions.

A-3. PROVISIONS FOR OTHER FACTORS

A-3.1 The initial reservoir level before the impact of the spillway design flood has to be taken as at full reservoir level. In regions experiencing prolonged floods where storms in quick succession are experienced, the spillway may also be checked for design flood preceded or succeeded by a flood of once in 25 years frequency. The interval between these two floods (peak to peak) may be taken as 3 or 5 days according to as the region lies in an annual rainfall zone of more than 100 cm or less than 100 cm respectively.

A-3.2 To provide for mechanical and other failures, it is assumed that some gates as inoperative with a maximum of 10 percent and a minimum of one gate. For this purpose the designer may be permitted to increase permissible stresses treating it as an extraordinary occurrence, like earthquake.

APPENDIX B

(*Clause* 5.2)

STEP-BY-STEP METHOD OF RESERVOIR FLOOD ROUTING FOR FIXING MAXIMUM WATER LEVEL (MODIFIED PULS' METHOD)

B-1. BASIC DATA REQUIRED

B-1.1 The following is the basic data:

   a) Reservoir level *versus* spillway capacity (outflow) (*see* Fig. 1), and storage capacity (*see* Fig. 1);

b) Full reservoir level;

c) Inflow hydrograph (*see* Fig. 2);

d) Initial outflow; and

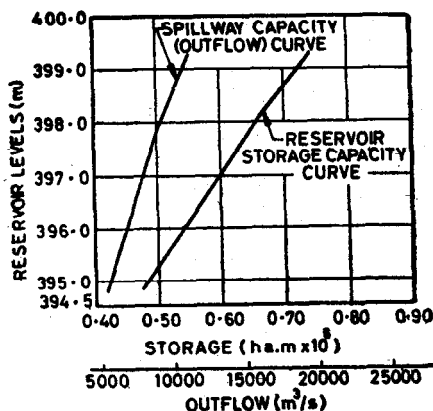e) Initial storage (or initial reservoir level).

FIG. 1   RESERVOIR LEVELS *Vs* SPILLWAY AND STORAGE CAPACITIES
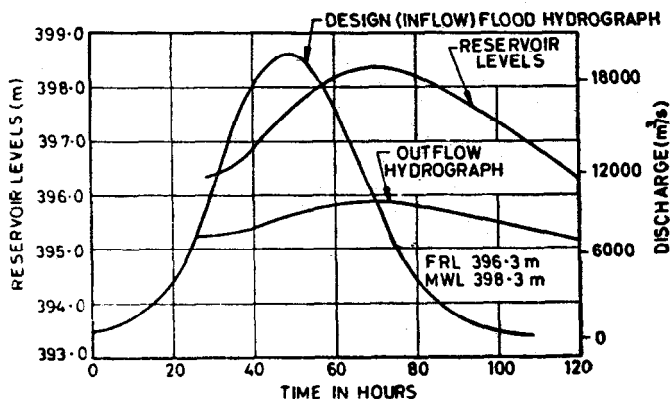
FIG. 2   INFLOW AND OUTFLOW HYDROGRAPHS, AND
TIME *Vs* RESERVOIR LEVELS FOR THE DESIGN FLOOD HYDROGRAPH

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B-2. BASIC EQUATION FOR ROUTING FLOOD THROUGH RESERVOIR

B-2.1 Basic equation for routing flood through reservoir is given below:

$$t\left(\frac{I_1 + I_2}{2}\right) - t\left(\frac{O_1 + O_2}{2}\right) = S_2 - S_1 = \triangle S \qquad \ldots (1)$$

where

t = time interval,

I_1 = initial inflow in m³/s,

I_2 = final inflow in m³/s after t h,

O_1 = initial outflow in m³/s,

O_2 = final outflow in m³/s after t h,

S_2 = final gross storage in cumec h after t h,

S_1 = initial gross storage in cumec h, and

$\triangle S$ = incremental storage in time t h.

Storage capacity is usually given in hectaremetres which should be converted into cumec h or cumec day, if the time interval t is expressed in hours or a part of a day.

B-3. PROCEDURE

B-3.1 Taking known values on one side

$$\left(\frac{I_1 + I_2}{2}\right) + \left(\frac{S_1}{t} - \frac{O_1}{2}\right) = \left(\frac{S_2}{t} + \frac{O_2}{2}\right) \qquad \ldots \qquad \ldots (2)$$

1 cumec day = 24 cumec h = 8·64 ha.m $\qquad \ldots \qquad \ldots (3)$

S ha.m $\quad = 2·771\ S$ cumec h $\qquad \ldots \qquad \ldots \qquad \ldots (4)$

$\dfrac{S}{t}$ ha.m $\quad = 2·771\ S/t$ cumec h $\qquad \ldots \qquad \ldots \qquad \ldots (5)$

Taking the time interval t as 6 h

$$\frac{S}{t} = 0·463\ S \qquad \ldots \qquad \ldots \qquad \ldots \qquad \ldots (6)$$

B-3.2 From Fig. 1, Table 1 is prepared.

B-3.3 By means of Table 1, Fig. 3 is prepared.

TABLE 1   RESERVOIR LEVEL Vs $\dfrac{S}{t} + \dfrac{O}{2}$

(ROUTING PERIOD — 6 HOURS)

(*Clause* B-3.2)

RESERVOIR LEVEL m)	STORAGE (S) (ha. m × 10⁶)	$\dfrac{S}{t} = 0.463\,S$ (cumec × 10⁶)	OUTFLOW (O) (cumec × 10⁶)	O/2 (cumec × 10⁶)	$\dfrac{S}{t} + \dfrac{O}{2}$ (cumec × 10⁶)
(1)	(2)	(3)	(4)	(5)	(6)
394·8	0·480 0	0·222 3	0·005 5	0·002 8	0·225 1
396·3	0·556 5	0·257 7	0·007 3	0·003 7	0·261 4
397·8	0·641 3	0·297 0	0·008 9	0·004 5	0·301 5
399·3	0·735 3	0·340 5	0·011 1	0·005 6	0·346 1

B-3.4 Referring to Table 2, col 1 and 2 are obtained from the given inflow hydrograph (Fig. 2). The values given in col 3 are the average of successive inflows. Column 4 is obtained by subtracting the initial $O/2$ value from the initial S/t value (which is either known or read out from Fig. 1 knowing the initial reservoir level).

Column 5 is the sum of col 3 and 4.

Column 6 is obtained by referring to Fig. 3 for the values of col 5.

Corresponding to the values of col 6, the values in col 7 are obtained by referring to Fig. 1.

With the values given in col 6 and 7, the outflow hydrograph and the curve of reservoir level with time are drawn in Fig. 2.
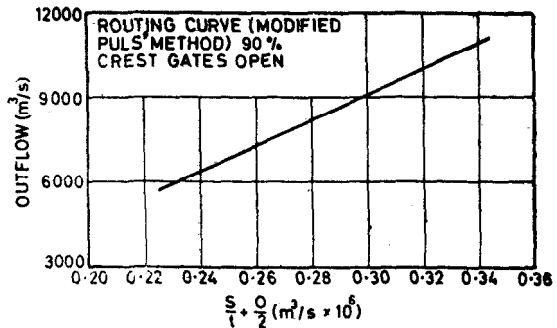

FIG. 3 $\dfrac{S}{t} + \dfrac{O}{2}$ Vs OUTFLOW CURVE

TABLE 2 ROUTING OF DESIGN FLOOD THROUGH RESERVOIR (ROUTING PERIOD — 6 HOURS)

(*Clause* B-3.4)

Time h	Inflow m³/s × 10⁶	Mean Inflow m³/s × 10⁶	$\left(\dfrac{S}{t} - \dfrac{O}{2}\right)$	$\left(\dfrac{S}{t} + \dfrac{O}{2}\right)$	Outflow m³/s × 10⁶	Reservoir Level m
(1)	(2)	(3)	(4)	(5)	(6)	(7)
26	0·007 1			0·261 4	0·007 1	396·30 FRL
		0·009 7	0·252 3			
32	0·012 3			0·262 0	0·007 3	396·42
		0·014 7	0·254 7			
38	0·017 1			0·269 4	0·007 6	396·76
		0·018 2	0·261 5			
44	0·019 3			0·279 7	0·008 1	397·14
		0·019 4	0·271 4			
50	0·019 6			0·290 8	0·008 6	397·59
		0·018 9	0·282 2			
	0·018 2			0·301 1	0·009 1	397·98
		0·016 5	0·291 9			
62	0·014 9			0·308 4	0·009 5	398·21
		0·013 0	0·299 0			
68	0·011 0			0·312 0	0·009 6	398·31 MWL
		0·009 2	0·302 4			
74	0·007 4			0·311 6	0·009 6	398·28
		0·005 7	0·302 1			
80	0·004 1			0·307 8	0·009 4	398·19
		0·003 2	0·298 4			
86	0·002 3			0·301 6	0·009 1	397·98
		0·001 7	0·289 5			
92	0·001 1			0·291 3	0·008 8	397·67
		0·000 9	0·285 2			
98	0·000 6			0·286 1	0·008 4	397·43
		0·000 6	0·277 7			
104	0·000 5			0·278 3	0·008 0	397·09
		0·000 5	0·270 3			
110	0·000 4			0·270 8	0·007 7	396·79
		0·000 4	0·263 0			
116	0·000 4			0·265 4	0·007 4	396·58
		0·000 4	0·256 4			
122	0·000 4			0·256 8	0·007 1	396·22

APPENDIX C

(*Clause* 5.2)

GRAPHICAL (SORENSEN'S) METHOD OF RESERVOIR FLOOD ROUTING FOR FIXING MAXIMUM WATER LEVEL

C-1. BASIC DATA REQUIRED

C-1.1 The following is the basic data:

a) Reservoir level *versus* spillway capacity (outflow) (*see* Fig. 4), and storage capacity (*see* Fig. 4);

b) Full reservoir level;

c) Inflow hydrograph (*see* Fig. 5A);

d) Initial outflow; and

e) Initial storage (or initial reservoir level).

C-2. BASIC EQUATION FOR ROUTING FLOOD THROUGH RESERVOIR

C-2.1 Basic equation for routing flood through reservoir is given below:

$$dS = (I - O)\, dt \qquad \ldots \qquad \ldots \qquad \ldots \qquad \ldots(7)$$

where

dS = change in reservoir storage,

I = rate of inflow,

O = spillway discharge rate (or outflow rate), and

dt = time interval.

As the inflow is a function of time and outflow is a function of storage, equation (7) may be written as:

$$S = S_f - S_i = \left[\bar{I} - \frac{(O_i + O_f)}{2} \right] \triangle t \qquad \ldots(8)$$

where $\bar{I}$ is the average inflow for the period $\triangle t$ and the subscripts 'i' and 'f' refer to the initial and final conditions of the period $\triangle t$.

C-3. PROCEDURE

C-3.1 For graphical solution, equation (8) is transposed as follows:

$$\left(S_i + \frac{O_i \triangle t}{2} \right) + \left(\bar{I} - O_i \right) \triangle t = S_f + O_f \frac{\triangle t}{2} \qquad \ldots(9)$$

14
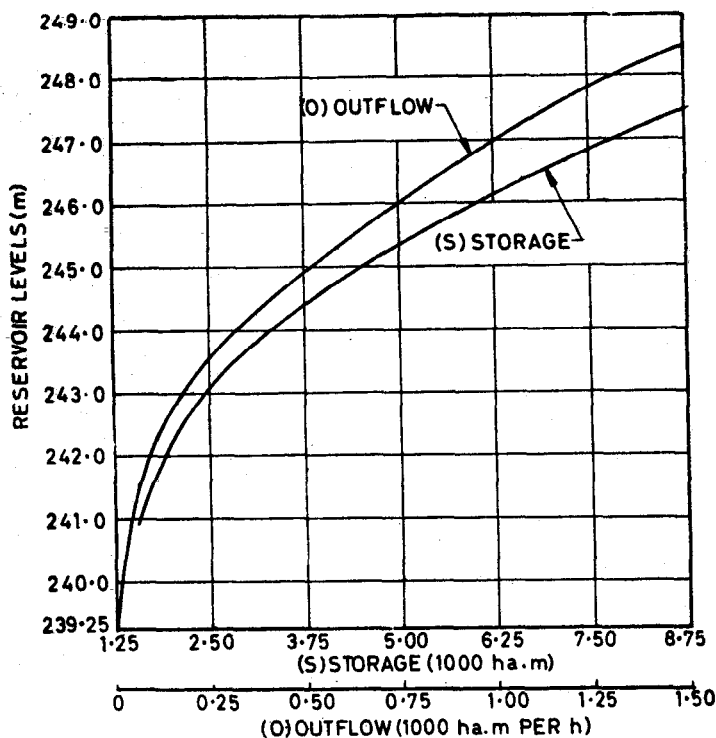
FIG. 4   RESERVOIR LEVEL *Vs* STORAGE AND OUTFLOW CURVES

A constant value of $\triangle t$ is selected, depending on the shape of the inflow hydrograph (here $\triangle t = 1$ h).   The values of $S + \dfrac{O \triangle t}{2}$ are plotted as a function of (a) reservoir level as shown in Fig. 5B, and (b) spillway discharge (or outflow) as shown in Fig. 5C.

NOTE — The scales for the ordinates of Fig. 5A and 5C should be the same.

In Fig. 5C by the side of $S + \dfrac{O \triangle t}{2}$ *versus* outflow curve, a line *A* is drawn with the same origin such that its slope is $\dfrac{C}{\triangle t}$ where *C* is the ratio of $S + \dfrac{O \triangle t}{2}$ scale to the *O* scale (that is, ratio of the scale of X-axis to the Y-axis of Fig. 5C).

For this particular example, $\triangle t$ is taken as one hour and the scale ratio works out to $\dfrac{1\,250}{250} = 5$, which determines the slope of the line A.

At 0800 h the reservoir inflow and the outflow are known. To determine the reservoir level one hour later, that is at 0900 h, the following procedure is adopted:

a) Draw a horizontal line a from the value of the average inflow $\overline{I}$ for the period 0800 h to 0900 h, towards Fig. 5C.

b) Draw a line b in Fig. 5C parallel to line A, from the intersection of horizontal (towards Fig. 5C) corresponding to the initial outflow at 0800 h and the curve of $S + \dfrac{O \triangle t}{2}$ *versus* O in Fig. 5C.

c) The intersection point of lines a and b is projected vertically in Fig. 5B and 5C to get the points Y and X respectively.

d) The point Y of Fig. 5B is projected horizontally to Fig. 5D to get the reservoir elevation at 0900 h.

e) The point X of Fig. 5C is projected horizontally to Fig. 5A to get the outflow O_f at 0900 h.

f) With new reservoir level and outflow values as initial conditions the steps a to e are repeated for the next hourly increment. Thus reservoir stage hydrograph Fig. 5D and the outflow hydrograph Fig. 5A (superimposed on the inflow hydrograph) are obtained.

The curves of Fig. 4 are not directly involved in the graphical solution but are the basic data for plotting Fig. 5B and 5C (*see* Table 3).

TABLE 3   COMPUTATION OF VALUES FOR Fig. 5B AND 5C
(WITH Δt AS 1 HOUR)

ELEVATION m	STORAGE, S 1 000 ha. m	OUTFLOW, O 1 000 ha. m/h	$\dfrac{O\Delta t}{2}$ 1 000 ha. m	$S + \dfrac{O\Delta t}{2}$ 1 000 ha. m
(1)	(2)	(3)	(4)	(5)
240·70	1·500	0·032	0·016	1·516
242·25	2·100	0·120	0·060	2·160
243·02	2·500	0·200	0·100	2·600
243·80	3·237	0·302	0·151	3·388
245·35	5·188	0·612	0·306	5·494
246·90	7·500	0·987	0·494	7·994

5D   Time *Vs* Reservoir Levels Curve

5B   Reservoir Levels *Vs* $S + \dfrac{O\Delta T}{2}$ Curve

h_f
h_i

$\Delta T = 1\,h$

T_i

RESERVOIR LEVELS (m)

$S_i + \dfrac{O_i \Delta T}{2}$

$S_f + \dfrac{O_f \Delta T}{2}$

y

TIME T(h)

AM   PM

(I) INFLOW

(O) OUTFLOW

$\dfrac{C}{\Delta T} = 5$

A

$\bar{I}$

a

$\bar{I} - O_i$

b

$\bar{I}$

O_f
O_i

T_i

$\Delta T = 1$

x

(I) INFLOW (1000 ha. m. PER h)
(O) OUTFLOW

TIME T (h)

AM   PM

5A   Inflow and Outflow Hydrographs

$S + \dfrac{O\Delta T}{2}$ (1000 ha. m)

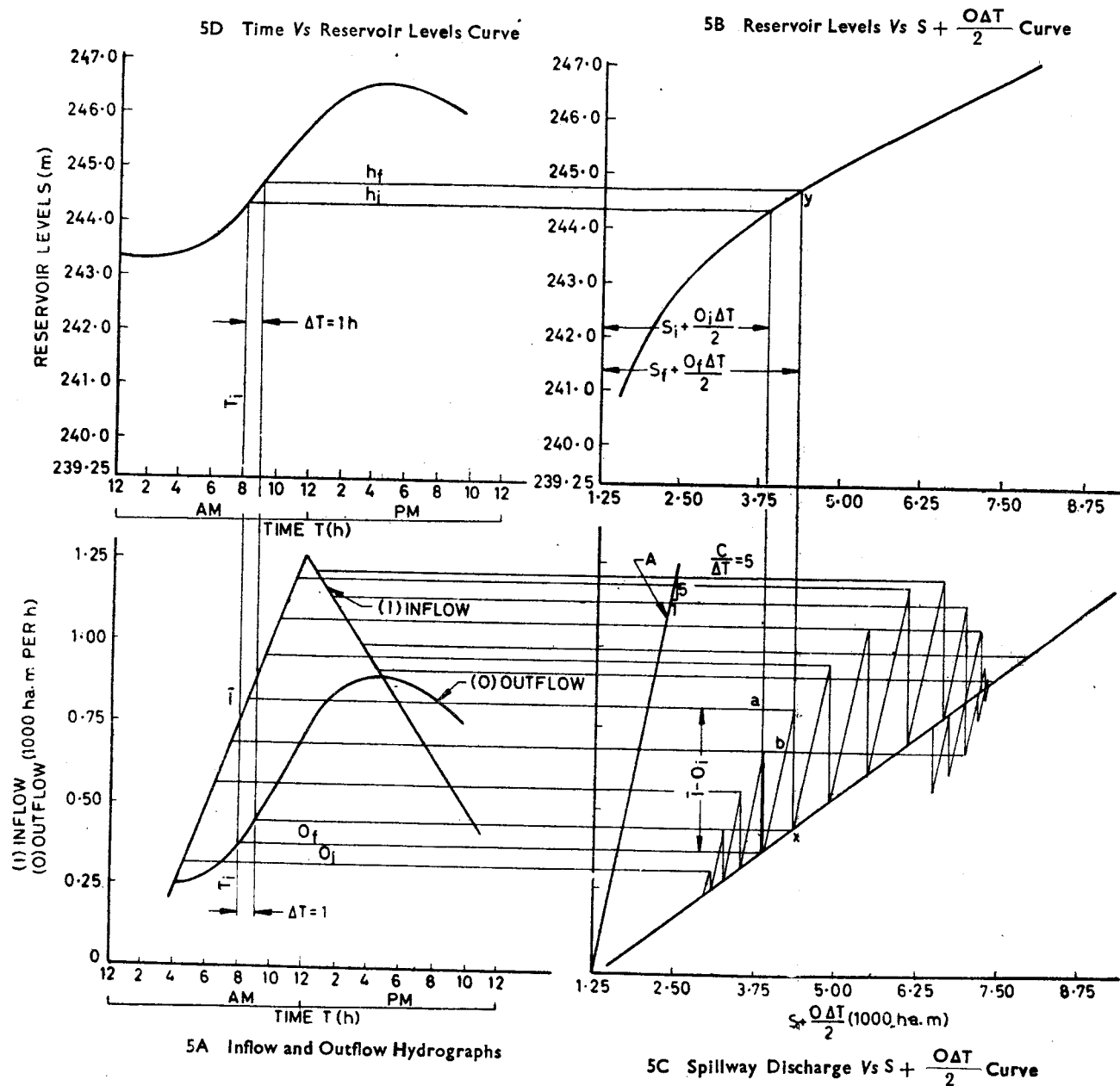5C   Spillway Discharge *Vs* $S + \dfrac{O\Delta T}{2}$ Curve

FIG. 5   GRAPHICAL (SORENSEN'S) METHOD FOR FIXING MAXIMUM WATER LEVEL

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APPENDIX D

(*Clause* A-2.2)

ESTIMATION OF PARAMETERS OF GUMBEL'S EXTREME VALUE DISTRIBUTION AND DETERMINATION OF PEAK FLOW OF LONG TERM RETURN PERIOD

D-1. MAGNITUDE OF PEAK FLOW OF A GIVEN RETURN PERIOD

D-1.1 Gumbel postulated the use of extreme value distribution

$$F(x)dx = ae^{-a(x-u)}e^{-e^{-a(x-u)}}dx \qquad \ldots \qquad \ldots(10)$$

where a and u are to be evaluated from n years annual observed peak flow values $x_1, x_2, x_3, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots, x_n$.

Peak flow magnitude x_T of T years return period is evaluated from

$$x_T = u - (1/a) \log_e \log_e T/(T-1)$$
$$= u - \frac{2\cdot302\,6}{a}\left[0\cdot362\,3 + \log_{10} \log_{10} \frac{T}{T-1}\right] \quad \ldots(11)$$

D-2. EXAMPLE

D-2.1 A worked out example for data of Sabarmati at Dharoi (1935-52), as given in Table 4 illustrates the computational procedure (1 m³/s = 1 000 l/s).

Referring to Table 4,

mean flood $= \bar{x} = \dfrac{\Sigma x}{n} = \dfrac{1\,672}{17} = 98\cdot352\,9 \qquad \ldots \qquad \ldots(12)$

Standard deviation s is given by

$$s^2 = \frac{\Sigma (x - \bar{x})^2}{n-1} = \frac{\Sigma x^2 - \dfrac{(\Sigma x)^2}{n}}{n-1} \qquad \ldots \qquad \ldots(13)$$

$$= \frac{256\,002 - 164\,446\cdot117\,6}{16} = 5\,722\cdot242\,650$$

$\therefore s = 75\cdot645\,506$

Computation for ' a '

As a first approximation to a, take

$$a_1 = \frac{\pi}{\sqrt{6}} \times \frac{1}{s} = 0\cdot016\,955 \qquad \ldots \qquad \ldots(14)$$

Then col 5 in Table 4 is completed. For a_2, a_3, a_4, etc, of col 6, 7 and 8 of Table 4, a step-by-step method of computing successive approximations a_{k+1} from a_k is given in Table 5, which is self-explanatory. Successive approximations are continued in Tables 4 and 5 till a negligibly small value approaching 0 of h_k is reached in Table 5.

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TABLE 4   VALUE OF e^{-ax}

(*Clause* D-2.1)

SL No.	YEAR	PEAK FLOW RATE (x) (10 000 l/s)	x^2	$e^{-a_1 x}$	$e^{-a_2 x}$	$e^{-a_3 x}$	$e^{-a_4 x}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	1935	76	5 776	0·275 7	0·263 1	0·262 4	0·262 4
2	1936	3	9	0·950 4	0·948 6	0·948 6	0·948 6
3	1937	125	15 625	0·120 1	0·111 3	0·110 8	0·110 8
4	1938	79	6 241	0·262 0	0·249 7	0·249 0	0·249 0
5	1939	227	51 529	0·021 3	0·018 6	0·018 4	0 018 4
6	1940	91	8 281	0·213 7	0·202 2	0·201 6	0·201 5
7	1941	42	1 764	0·490 6	0·478 2	0·477 4	0·477 4
8	1942	125	15 625	0·120 1	0·111 3	0·110 8	0·110 8
9	1943	193	37 249	0·037 9	0·033 7	0·033 5	0·033 5
10	1944	246	60 516	0·015 4	0·013 3	0·013 2	0·013 2
11	1945	187	34 969	0·042 0	0·037 5	0·037 2	0·037 2
12	1946	45	2 025	0·466 2	0·453 6	0·452 9	0·452 9
13	1947	57	3 249	0·380 5	0·367 5	0·366 7	0·366 7
14	1948	20	400	0·712 4	0·703 7	0·703 3	0·703 3
15	1949	48	2 304	0·443 2	0·430 3	0·429 6	0·429 6
16	1951	102	10 404	0·177 4	0·166 6	0·166 1	0·166 1
17	1952	6	36	0·903 3	0·899 9	0·899 7	0·899 7
	TOTAL	1 672	256 002	5·632 2	5·489 1	5 481 2	5·481 1

Computation for 'u'

From the steady value so reached of a_k for $k = 4$ (in the present case), compute

$$u = \log_e 10 \times \frac{1}{a_k} \left[\log_{10} (n) - \log_{10} (\Sigma e^{-a_k x_i}) \right] \qquad \ldots(15)$$

$$= 64 \cdot 297\ 104$$

Putting these values of u and the steady a_k in (11), expected peak values x_T for $T = 50; 100; 200; 500$, etc, years return periods are evaluated as

$$x_{50} = 285 \cdot 97; \quad x_{100} = 325 \cdot 63; \quad x_{200} = 365 \cdot 15; \quad x_{500} = 417 \cdot 30$$

In order to safeguard against incidental errors of estimation arising from small samples of 29, 30, ...years observed data available the above best estimates may be increased by $1 \cdot 645\ SE(x_T)$ to warrant 95 percent dependability of the design estimates. The standard error of

estimation $SE(x_T)$ is given by

$$SE(x_T) = \frac{1}{a\sqrt{n}}\left[1 + \frac{6}{\pi^2}\left\{1 - 0.577\ 216\right.\right.$$
$$\left.\left. - \log_e\ \log_e T/(T-1)\right\}^2\right]^{\frac{1}{2}} \qquad \ldots \qquad \ldots (16)$$

$$= \frac{1}{a\sqrt{n}}\left[1 + 0.607\ 927\ 1\left\{0.422\ 784\right.\right.$$
$$\left.\left. - \log_e\ \log_e T/(T-1)\right\}^2\right]^{\frac{1}{2}} \qquad \ldots \qquad \ldots (17)$$

Thus

$$x_{50} = 2\ 859\ 700\ \ \text{l/s}\ (= 2\ 859.7\ \text{m}^3/\text{s}) + 797.3\ \text{m}^3/\text{s}$$
$$x_{100} = 3\ 256\ 300\ \ ,,\ \ (= 3\ 256.3\ \text{m}^3/\text{s}) + 916.1\ \text{m}^3/\text{s}$$
$$x_{200} = 3\ 651\ 500\ \ ,,\ \ (= 3\ 651.5\ \text{m}^3/\text{s}) + 1\ 035.7\ \text{m}^3/\text{s}$$
$$x_{500} = 4\ 173\ 000\ \ ,,\ \ (= 4\ 173.0\ \text{m}^3/\text{s}) + 1\ 194.6\ \text{m}^3/\text{s}$$

Values of $A = -\log_e\ \log_e T/(T-1)$ for the most commonly used values of T like 20, 50, 100, 200, 500, and 1 000 years are reproduced below for convenience of users:

$T =$	20	50	100	200	500	1 000
$A =$	2.9702	3.9020	4.6002	5.2958	6.2137	6.9073

D-2.2 It is advisable to procure always at least 20-25 years observed records and not to compute estimates for return periods longer than about $20n$ or $25n$ years.

D-2.3 The steps remain very much the same irrespective of the value of n, the number of years. Though the computational steps by successive approximation method appear very arduous, their schematically outlined procedure as in **D-2.1**, even with recorded data of around 50 years should not take a person equipped with an ordinary calculating machine and a book of logarithm tables more than 6-7 hours. With electronic fast computers the stabilized expression can be secured in two to three minutes for which programme is given in **D-3**.

D-2.4 The calculation of a 500 years' flood, for example does not tell when the flood is coming; it might occur in any year within that period or not until 500 years have elapsed.

D-2.5 The choice of a suitable return period on which to base the 'design flood' is the engineer's responsibility, and rests ultimately on his judgement and experience. The size and cost of the dam, the design freeboard, the amount of water stored, and the likely consequences of failure are factors which influence the selection.

TABLE 5 FOR VALUES OF a_{k+1} FROM a_k

(*Clause* D-2.1)

Step	Quantity	$k = 1$	$k = 2$	$k = 3$	$k = 4$
1	a_k	0·016 955	0·017 565	0·017 602	0·017 602 58
2	$a_k \log_{10} e$	0·007 363	0·007 628	0·007 644	0·007 645
3	$1/a_k$	58·979 6	56·931 4	56·811 7	56·809 9
4	$1/a^2{}_k$	3 478·593 2	3 241·184 3	3 227·569 3	3 227·364 7
5	$\bar{x} - 1/a_k$	39·373 3	41·421 5	41·541 2	41·543 0
6	$\Sigma e^{-a_k x_i}$	5·632 2	5·489 1	5·481 2	5·481 1
7	$\Sigma x_i e^{-a_k x_i}$	240·075 2	228·368 4	227·711 1	227·702 0
8	$\Sigma x^2{}_i e^{-a_k x_i}$	19 898·861 8	18 563·741 2	18 488·027 9	18 487·199 8
9	$f(a_k) = (7) - (5)(6)$	18·316 900	1·001 644	0·015 475	0·000 663
10	$f'(a_k) = (8) + (5)(7) - (4)(6)$	−30 038·441 5	−26 895·564 3	−26 719·588 4	−26 717·284 3
11	$h_k = -\dfrac{f(a_k)}{f'(a_k)}$	0·000 610	0·000 037	0·000 000 58	0·000 000 02
12	$a_{k+1} = a_k + h_k$	0·017 565	0·017 602	0·017 602 58	0·176 026 0

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D-3. COMPUTER PROGRAMME FOR PEAK FLOOD ESTIMATION BY MAXIMUM LIKELIHOOD METHOD

```
FIRST DATA CARD IS NO.  OF OBSERVATION 4COLS, NAMES OF RIVER,
SITE, UNIT OF OBSERVATION 8COLS EACH FOLLOWED BY DATA CARDS
4COLS EACH OBSERVATION.
      DIMENSION X( 100 ), EX( 100 )
      SUM = 0.
      SQSUM=0.
      READ 2, N, RVR, SITE, UNIT
2  FORMAT ( 14, 3A8 )
      READ3, ( X (I), I=1, N )
3  FORMAT (20F4.2)
      DO 5 I=1, N
      SUM=SUM+X(I)
5  SQSUM=SQSUM+X(I)**2
      B=N
      SQSUM=(SQSUM—SUM*SUM/B)/(B-1.)
      SUM=SUM/B
      T=3.141592654
      A=T/SQRTF(6.*SQSUM)
      H=0.
      DO 1 J=1, 20
      SIG=0.
      XSIG=0.
      X2SIG=0.
      A=A—H
      DO 8 I=1, N
      EX( I ) = EXPF ( — A*X( I ) )
      SIG = SIG + EX( I )
      XSIG = XSIG + X( I )*EX( I )
8  X2SIG = X2SIG + X( I )*X( I )*EX( I )
      FA = XSIG — ( SUM — 1./A )*SIG
      FH = ( SUM — 1./A )*XSIG — X2SIG — SIG/A**2
      H = FA/FH
      IF ( ABSF ( H ) — 1.E — 06 ) 30, 30, 1
1  CONTINUE
30  PIE = 6./( T**2 )
      T = 1./A
      U = T*LOGF( B/SIG )
      X20 = U + T*2.970186
      X50 = U + T*3.901953
      X100 = U + T*4.600150
      X200 = U + T*5.295775
      X500 = U + T*6.213675
      P = 1./( A*SQRTF ( B ) )
      SEX20 = P*SQRTF ( 1. + PIE*( 0.422784 + 2.970186 )**2 )
      SEX50 = P*SQRTF ( 1. + PIE*( 0.422784 + 3.901953 )**2 )
      SEX100 = P*SQRTF ( 1. + PIE*( 0.422784 + 4.600150 )**2 )
      SEX200 = P*SQRTF ( 1. + PIE*( 0.422784 + 5.295775 )**2 )
      SEX500 = P*SQRTF ( 1. + PIE*( 0.422784 + 6.213675 )**2 )
      PRINT 310, UNIT, RVR, SITE
```

(Continued)

```
310    FORMAT(1H1/////30X 43HEXPECTED VALUES WITH VARIOUS RETURN PERIO
      1 46X, 1H*, A8, 4X, 1H*//40X, A8,3X,2HAT,A8 )
       PRINT 309
309     FORMAT( //29X, 8H20 YEARS, 12X, 8H50 YEARS, 11X, 9H100 YEARS, 11X, 9H20
      1 EARS, 11X, 9H500 YEARS )
       PRINT 311, X20, X50, X100, X200, X500
311     FORMAT( 8X, 9HESTIMATE =, 5 ( 4X, F16.9 ) )
       PRINT 312, SEX20, SEX50, SEX100, SEX200, SEX500
312     FORMAT( /9X, 8HSTD ERR =, 5 ( 4X, F16.9 ) )
       STOP
       END
```

INTERNATIONAL SYSTEM OF UNITS (SI UNITS)

Base Units

QUANTITY	UNIT	SYMBOL
Length	metre	m
Mass	kilogram	kg
Time	second	s
Electric current	ampere	A
Thermodynamic temperature	kelvin	K
Luminous intensity	candela	cd
Amount of substance	mole	mol

Supplementary Units

QUANTITY	UNIT	SYMBOL
Plane angle	radian	rad
Solid angle	steradian	sr

Derived Units

QUANTITY	UNIT	SYMBOL	DEFINITION
Force	newton	N	$1\ N = 1\ kg.m/s^2$
Energy	joule	J	$1\ J = 1\ N.m$
Power	watt	W	$1\ W = 1\ J/s$
Flux	weber	Wb	$1\ Wb = 1\ V.s$
Flux density	tesla	T	$1\ T = 1\ Wb/m^2$
Frequency	hertz	Hz	$1\ Hz = 1\ c/s\ (s^{-1})$
Electric conductance	siemens	S	$1\ S = 1\ A/V$
Electromotive force	volt	V	$1\ V = 1\ W/A$
Pressure, stress	pascal	Pa	$1\ Pa = 1\ N/m^2$

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